LABORATORY MODEL TESTING ON SUCTION PILES AND ANALYTICAL SIMULATION OF INSTALLATION

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ABSTRACT

As part of the feasibility study of the Mobile Offshore Bases, suction piles are currently being studied to provide the necessary mooring capability. This paper presents the preliminary results of the experimental laboratory model tests on suction piles in sand. The results have been used to calibrate the mobilized soil friction angle included in the analytical simulation of the suction pile installation, i.e., the suction pressure vs. pile penetration relationship. The mobilized soil friction angle is described as a function of the total head gradient between the pile tip and the inside soil surface.

En partie d'une étude de possibilité des fondements mobiles au large, on étudie actuellement des pilotis aspirants pour pouvoir la capabilité nécessaire pour s'amarrer. Cette étude présente les résultats préliminaires des épreuves modèles laboratoires expérimentales sur des pilotis aspirants dans du sable. On a employé les résultats pour calibrer l'angle de frottement du terrain mobilisé compris dans la simulation analytique de l'installation du pilotis aspirants, c'est-à-dire le rapport de la pression d'aspiration contre la pénétration des pilotis. L'angle de frottement du terrain mobilisé est décrit comme une fonction de l'inclinaison de l'énergie totale entre l'extrémité du pilotis et l'intérieur de la surface du terrain.

1. INTRODUCTION

The US Office of Naval Research is currently conducting a feasibility study program to advance critical design technologies for Mobile Offshore Bases (MOB). The MOB is expected to be a self-propelled, floating, prepositioned base that could move for long-term deployment. The dimensions of the MOB are approximately 1,500 meters by 120 meters with the internal storage space of 800,000 m^2.

The South Dakota School of Mines and Technology participates in the MOB feasibility study to provide an adequate mooring technique for this very large floating structure. Since the vertical and lateral loads expected from the MOB is to be extremely large in magnitude, any conventional underwater mooring technique may not provide adequate resistance. For this reason, suction piles that have been introduced recently are currently being investigated to identify whether they can provide the necessary mooring capability.

A suction pile typically has a large diameter (up to 32 meters have been used to date) with a relatively small length to diameter ratio. It is installed by applying a suction pressure inside the pile, which results in a net external surcharge that pushes the pile into the seafloor. The details of the suction pile with regard to it use, mechanism, installation, and analysis and design methods can be found in references (Burgess et al. 1981, Burgess and Hird 1983, Hogervost 1980, Morrison and Clukey 1994, Senpere and Auvergne 1982, Tjelta et al. 1986).

This paper describes the details of the laboratory experimental model tests on suction piles in sandy seafloor. The test facility, test details, results, and calibration of the analytical solution method are included.

2. TEST FACILITY

2.1 Model Test Tank

The equipment used for the pile installation tests is shown schematically in Figure 1. The test tank consisted of two sections of a 61 cm diameter heavy duty PVC water pipe. The lower section was made from the flange of the pipe and was attached with turnbuckles to a one half inch thick steel plate. In order to provide a water-proof seal, a one half inch thick neoprene sheet was placed between the steel plate and the pipe flange. For backwashing and drainage purpose, a perforated steel plate was installed inside the pipe above the flange. The perforated plate was covered with a geofabric to prevent sand from washing into the backwash chamber. A 150 cm straight pipe section was then installed into the top portion of the flange and sealed with a standard pipe seal.

Prior to testing, the tank was filled with water, and sand was pluviated into the tank. The sand was placed to within 15 cm of the top of the pipe. The density of sand was kept constant by maintaining the height of the sand column at the same height at the beginning of each test through a combination of backwashing and agitation. Backwashing

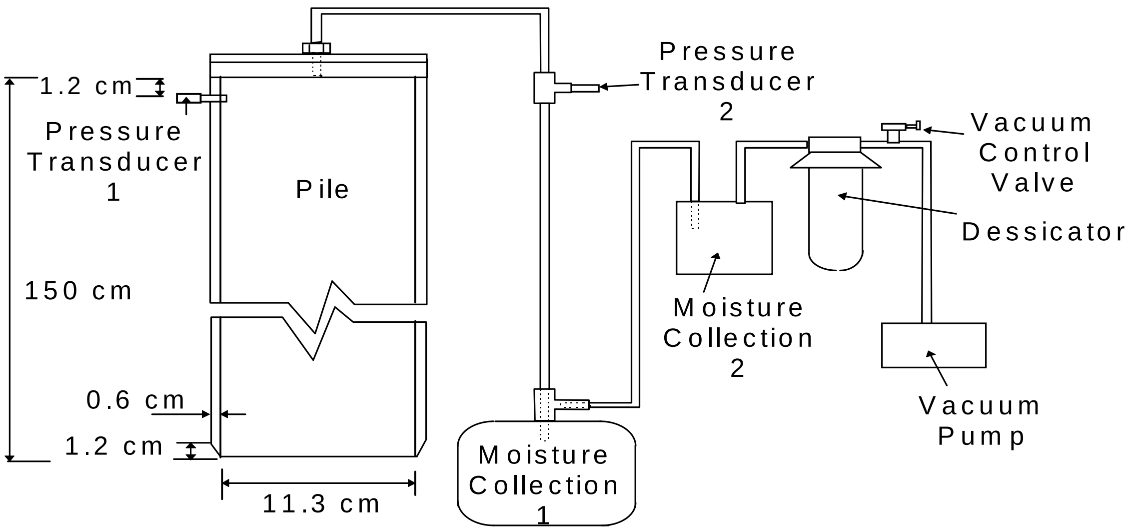

Figure 1. Schematics of Pile and Vacuum System

was done by applying a high water pressure through the chamber below the soil column and agitation was done with a perforated plastic pipe with a high water pressure.

2.2 Model Pile

The model pile consisted of a 12.57 cm outside diameter and 150 cm long Plexiglas pipe with a wall thickness of 0.6 cm. The tip of the pile was beveled at an angle of approximately 30 degrees with the longitudinal axis of the pile. The top of the pile was capped with a Plexiglas disk and a vacuum pipe leading to a vacuum pump was attached through the center of the plate. Pressure transducers to record the level of vacuum were attached near the top of the pile and on the pipe outside the pile. The vacuum pipe was equipped with two moisture collection chambers and a dessiccator chamber to minimize the amount of moisture entering the vacuum pump. A vacuum control valve was attached to the pipe between the dessiccator chamber and the vacuum pump.

Penetration of the pile was measured with a roller wheel assembly connected to a potentiometer. The assembly was fixed to the top of the model tank. The roller wheel was placed against the wall of the pipe. In order to eliminate possible slippage, a narrow sand paper strip along the pile outside surface was attached to improve the roller friction.

2.3 Soil Properties

The soil used for the experiments consisted of fine sand obtained commercially and used for sand blasting. The material is a sub-rounded, poorly graded sand with 100 percent passing the number 16 and less than 1 percent passing the number 100 and 200 US standard sieves.

3. EXPERIMENT DETAILS

3.1 Pile Installation

Prior to installation of the pile, the sand was backwashed and agitated to raise its surface to a preset level corresponding to a dry unit weight of 17 kN/m^3. The pile was then manually pushed into the sand to a preset initial penetration depth, and vacuum was applied and gradually increased until movement started. Extreme care was taken not to create a quick condition, as this would fill up the pile with sand quickly and prevent further penetration.

3.2 Test Procedure

After the pile was seated, all instruments were set to zero readings on the digital data acquisition system (DAS), and the soil and water column heights rechecked. Vacuum was then carefully applied in increments by opening and adjusting the vacuum control valve (Figure 1) until the pile started to move. The vacuum level was then maintained at that level until movement ceased. As long as the pile moved, manual pressure readings were taken at every 3.2 mm (1/8 inches). DAS increments were set at 10 second intervals. Pile movement generally occurred in increments of 3.2 to 6.4 mm (1/8 to 1/4 inches) or less. Water level readings both inside and outside were taken every third pile movement. By carefully adjusting the pressure level inside the pile, quick condition was prevented, however the soil column inside the pile was observed to rise slightly. It was felt that this was due to both displacement of the sand caused by the pile penetration and the upward water flow gradient in the pile. The soil column level was recorded manually.

As the pile penetration progressed under constant vacuum level, the time interval between movement increments increased. If the time interval between increments reached five minutes or longer, the vacuum pressure inside the pile was increased gradually until the movement again commenced, and the procedure repeated until the pile was fully penetrated or until a quick condition inside the pile occurred.

4. RESULTS

Tests were conducted in three series. The first test series comprised of five identical tests with a surcharge load of 0.17 kN mounted on top of the pile (Series 1 A through 1 E). The second test series (Series 2A through 2F) comprised of six tests with a surcharge load of 0.24 kN, while the third series comprised of four tests (Series 3A through 3D) with a surcharge load of 0.28 kN. These surcharge loads are additional weights that are equivalent to 100%, 125% and 150% of the weight of a steel pile with the same dimensions. All tests had an initial penetration depth of pile of approximately 0.3 meters. The test results are shown in Figures 2 through 4 in terms of the pile penetration vs. applied pressure below the atmospheric pressure. Some data scatter from test to test exists, especially with Test Series 1. Test 1A was the first test run and readings were only taken every 2.5 cm. Penetration was also limited to a total of 45.7 cm. For Test Series 2 and 3, initial penetration was measured carefully to within 2.5 mm.

The curves in the figures show a slightly concave shape, indicating that the average pile resistance decreases at a faster rate with depth. This is in contrast to the general pile bearing capacity theory. This contradiction however can be explained with the reduction of soil internal friction angle due to the upward flow of water within the pile. As the suction pressure is applied, a steady state water flow from the soil surface outside the pile through the pile tip to the inside soil surface. The downward water flow outside the pile would not influence the soil strength characteristic much, whereas the lateral flow of water at the pile tip and the upward flow of water inside the pile would significantly reduce the soil strength characteristic, resulting in much reduced end bearing at the pile tip and skin friction along the inside surface of the pile.

The average vacuum pressure vs. total penetration relationship for each test series is shown in Figure 5, which also shows a slightly concave relationship between the pressure and penetration. It is also clearly seen that as the surcharge weight increased, the necessary vacuum pressure necessary to move the pile decreases, almost in direct proportion to the surcharge weight.

5. ANALYTICAL SIMULATION OF PILE INSTALLATION

As explained in the previous section, the steady state water flow created by the applied suction pressure definitely influences the soil strength characteristic. It is therefore reasonable to assume that the developed soil internal friction angle is not influenced along the outer surface of the pile, while that is somewhat influenced at the pile tip and within the pile. The degree of influence would vary at various points of the pile, depending on the intensity of water flow. Since it is virtually impossible to measure the degree of influence at every depth, it is conceived to introduce an average value, i.e., the “mobilized soil internal friction angle”, to represent the overall behavior of the soil strength influenced by the water flow with a single term. It is assumed to be a function of the total head gradient between the pile tip and the inside soil surface.

The steps involved in the analytical simulation of the pile installation is explained below.

1) With known surcharge weight and the pile weight, the pile initial penetration depth is calculated by equating the total weight with the soil bearing capacity that is depth dependent. At this step, the mobilized soil internal friction angle is equal to the soil internal friction angle since no water flow occurs.
2) With a known initial pile penetration depth, the maximum suction pressure that can be applied inside the pile (critical suction pressure) is calculated. The maximum suction pressure is the pressure that creates a quick condition of sand inside the pile.
3) Suction pressure whose magnitude is less than the critical suction pressure is applied and a steady state, confined seepage problem is solved with known boundary conditions.
4) The average mobilized soil internal friction angle ϕ_m is estimated from the α vs. i relationship, where α is the ratio between $\tan \phi_m$ and $\tan \phi$, and i is the total head gradient between the pile tip and the inside soil surface.
5) The pile bearing capacity at a new pile penetration depth is iteratively solved with the soil internal friction angle being ϕ at the outside surface and ϕ_m at the pile tip and the inside surface.
6) Steps (2) through (5) are repeated until the pile does not penetrate any further, or the critical suction pressure is reached, or the entire length of the pile is penetrated.

The pile bearing capacity at any given depth, Q, is obtained

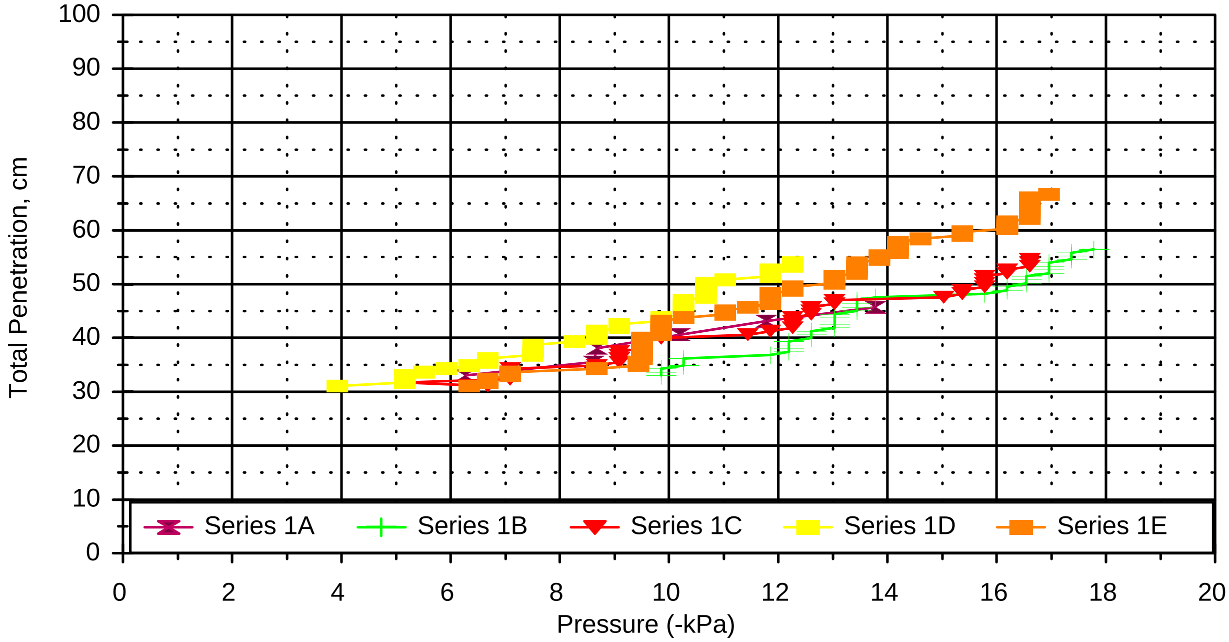

Figure 2. Pressure vs Penetration Test Series 1

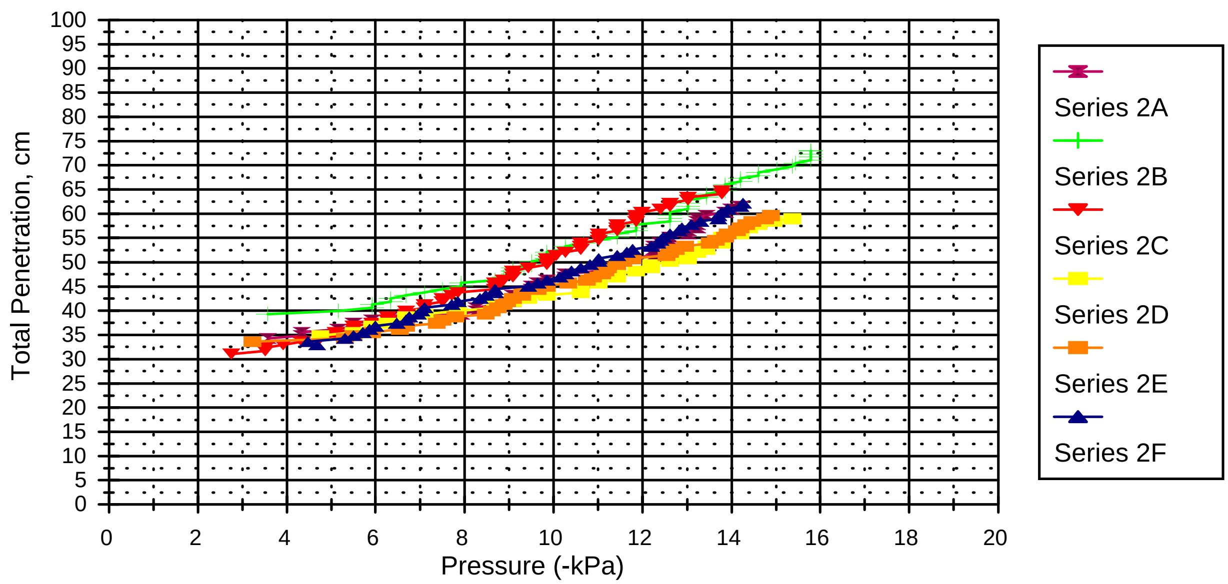

Figure 3. Pressure vs Penetration Test Series 2

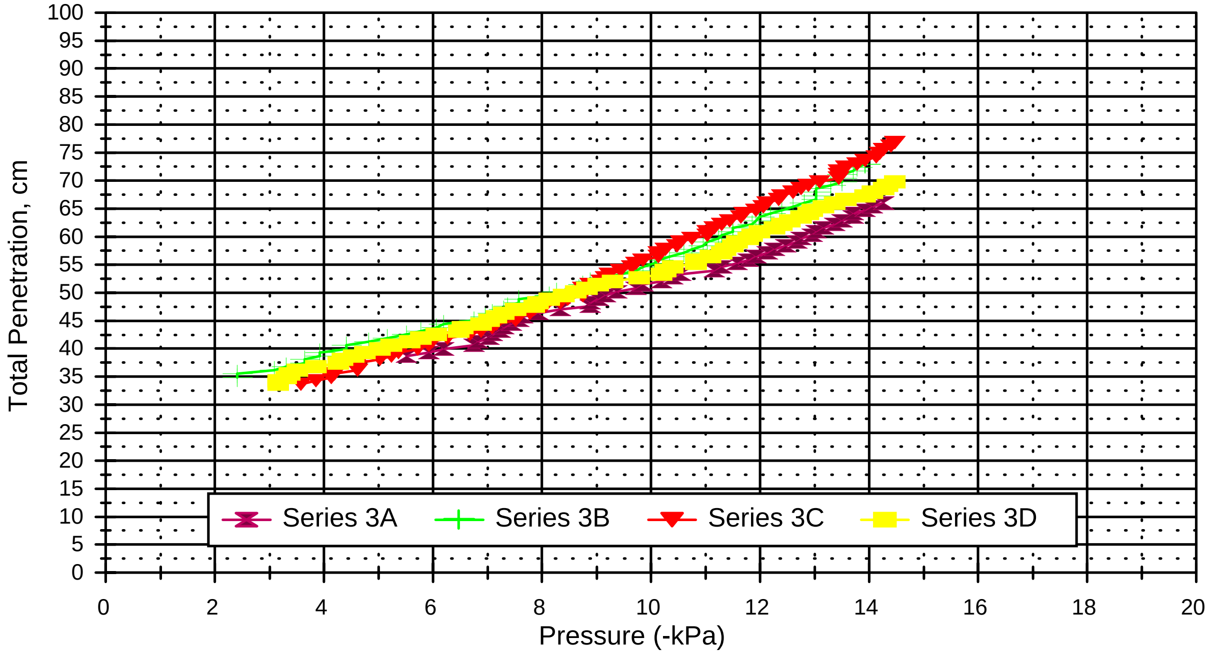

Figure 4. Pressure vs Penetration Test Series 3

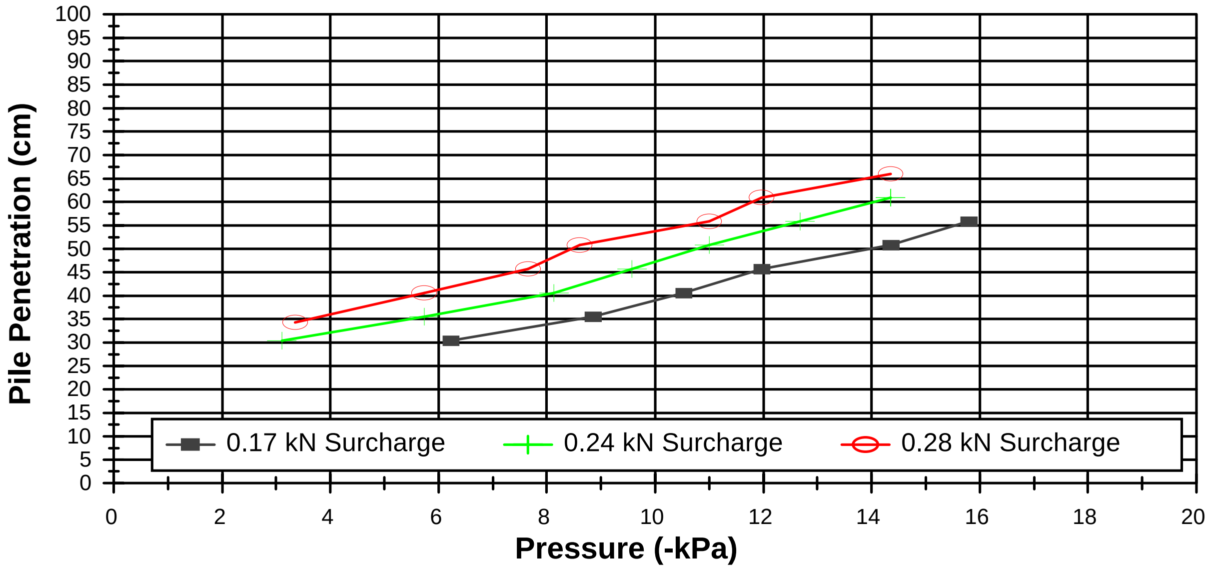

Figure 5 . Pressure vs Penetration Summary

from the smaller of Q_1 and Q_2, where Q_1 is the bearing capacity assuming that the soil inside the pile moves as a unit with the pile and Q_2 is the bearing capacity assuming that the soil inside the pile moves independent to the pile. Q_1 and Q_2 therefore can be estimated from

$$Q_1 = Q_{gross,\,tip} + Q_{outside} - W_{pile} - W_{soil} \qquad [1]$$

$$Q_2 = Q_{net,\,tip} + Q_{outside} + Q_{inside} - W_{pile} \qquad [2]$$

where

$Q_{gross,\,tip}$ = tip bearing capacity with the gross pile tip area based on ϕ_m

$Q_{net,\,tip}$ = tip bearing capacity with the net pile tip area based on ϕ_m

$Q_{outside}$ = outside skin friction based on soil friction angle of ϕ

Q_{inside} = Inside skin friction based on soil friction angle of ϕ_m

W_{pile} = buoyant weight of pile

W_{soil} = buoyant weight of soil inside the pile.

The conventional shallow foundation bearing capacity equation was used for the tip bearing capacity with depth and shape factors as suggested by Hansen (1970) and De Beer (1970), respectively. The skin friction was calculated based on the at-rest lateral earth pressure condition with the wall friction coefficient being one half of the soil internal friction coefficient.

6. CALIBRATION OF MOBILIZED SOIL FRICTION ANGLE

The experimental test results described previously have been used to calibrate the value of α, i.e., the ratio between $\tan \phi_m$ and $\tan \phi$ by equating the applied surcharge (suction pressure and the static weight above the pile) with the calculated bearing capacity at the measured pile penetration depth. The values of α have been calculated at every measured pile depth as shown in Figure 6. As can be seen from the figure, α decreases as the pile penetration increases.

As discussed previously, the value of α is thought to be related directly with the total head gradient between the pile tip and the inside soil surfaces, since the intensity of the upward flow of water dictates the soil denseness, therefore the soil strength characteristic.

The calculated values of α therefore have been replotted as a function of the total head gradient as shown in Figure 7. As expected, the value of α gradually decreases with increasing total head gradient (i), at an almost linear fashion. In theory, the value of α should be 1.0 at i = 0, i.e., no water flow. However, the calibrated value of α as shown in the figure may be greater than 1.0 at i less than 0.4. This is because the adopted bearing capacity equation is not completely perfect in estimating the soil resistance. As a matter of fact, no two bearing capacity equations yield identical results. Therefore, the value of α should be viewed as method-dependent as well as a function of the water flow.

The value of α does not approach to zero even at a value of i high enough to cause a quick condition in soil. The calibrated value of α is about 0.8 at the value of i = 1.2. This is due to the definition of the mobilized soil friction angle in this paper. It is defined as the average soil friction angle between the pile tip and the inside soil surface as influenced by the water flow. Even when the soil inside the pile is in quick condition, the soil near the tip of the pile is not necessarily in quick condition. The average mobilized soil friction angle therefore could be above zero.

7. CONCLUSIONS

Experimental laboratory model tests on suction piles have been conducted to provide calibration necessary for establishing an analytical solution between the suction pile penetration in sandy seafloor and the applied suction pressure inside the pile. Details on the experiment setup, test procedures, test results, and the calibration of the mobilized soil internal friction angle are described.

Experiments indicate that suction is very effective in penetrating piles even in sand. However, the applied suction is limited due to the possible soil instability within the pile, i.e., quick condition. It was also observed that the state of soil within the pile became sufficiently loose due to the upward water flow caused by the applied suction pressure. Therefore, the conventional bearing capacity equations may not be used directly. Mobilized soil friction angle was therefore introduced to describe the average reduction in soil internal friction angle between the pile tip and the inside soil surface.

An analytical solution method was also formulated to establish the relationship between the pile penetration and the applied suction pressure, using the mobilized soil friction angle. The mobilized soil friction angle was expressed as a function of the total head gradient between the pile tip and the inside soil surface. The experimental results were then used to calibrate the mobilized soil friction angle.

The calculated values of the mobilized soil friction angle are found to be very reasonable. However, additional experiments must be conducted in order to identify additional factors that may influence the value of the mobilized soil friction angle, such as the pile diameter to length ratio, the soil characteristics, and the initial pile penetration depth.

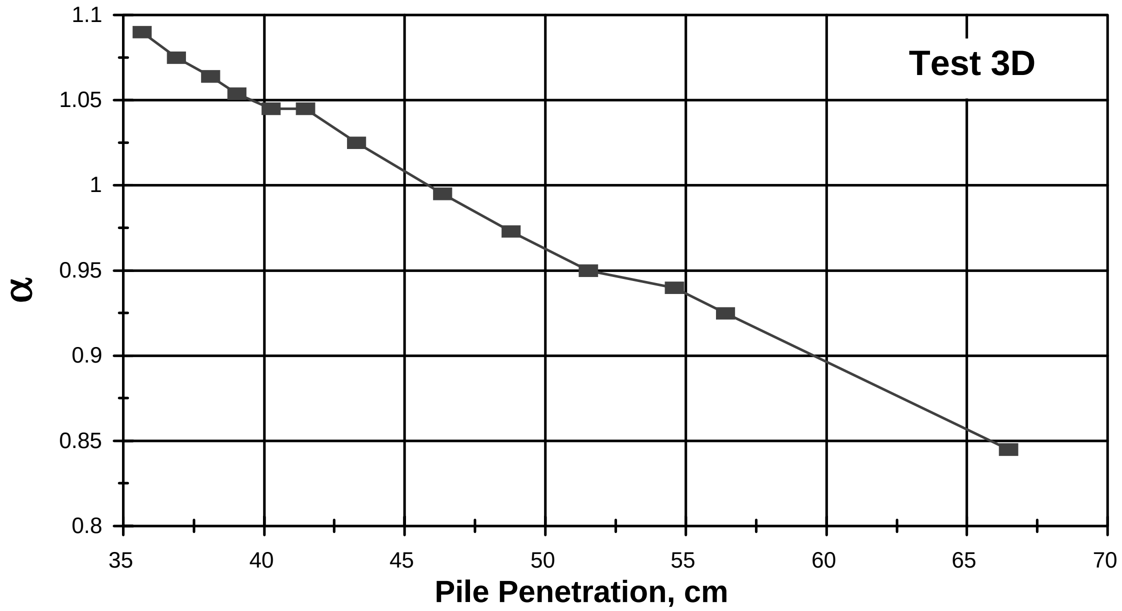

Figure 6. Alpha vs Pile Penetration

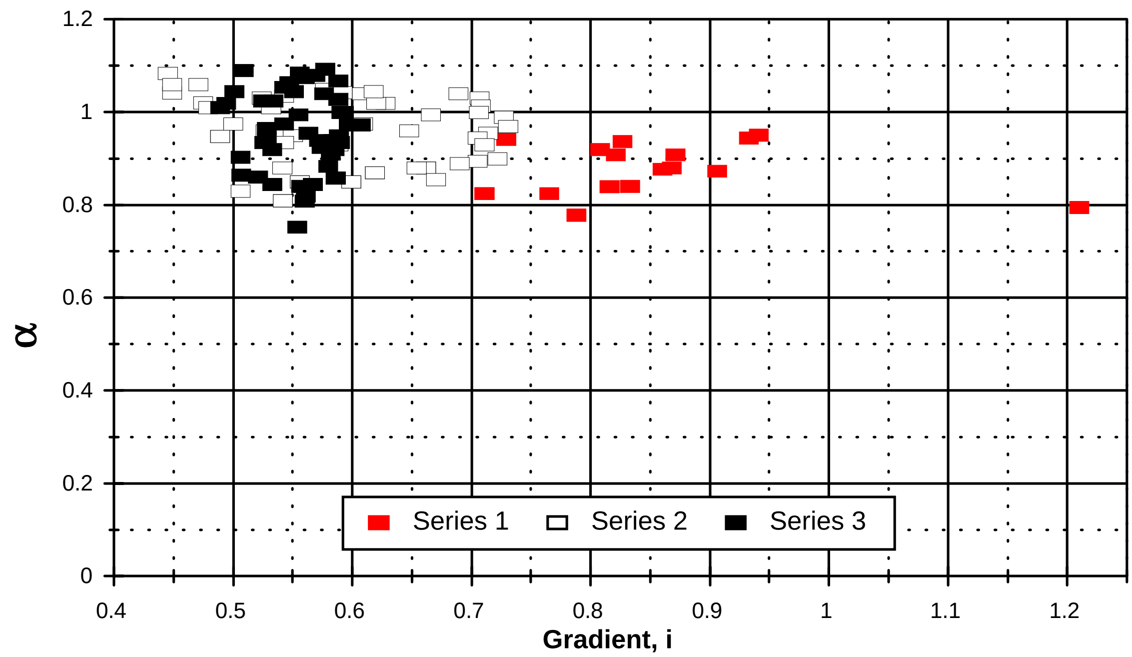

Figure 7. Alpha vs Gradient
Summary of all Series

8. ACKNOWLEDGMENTS

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